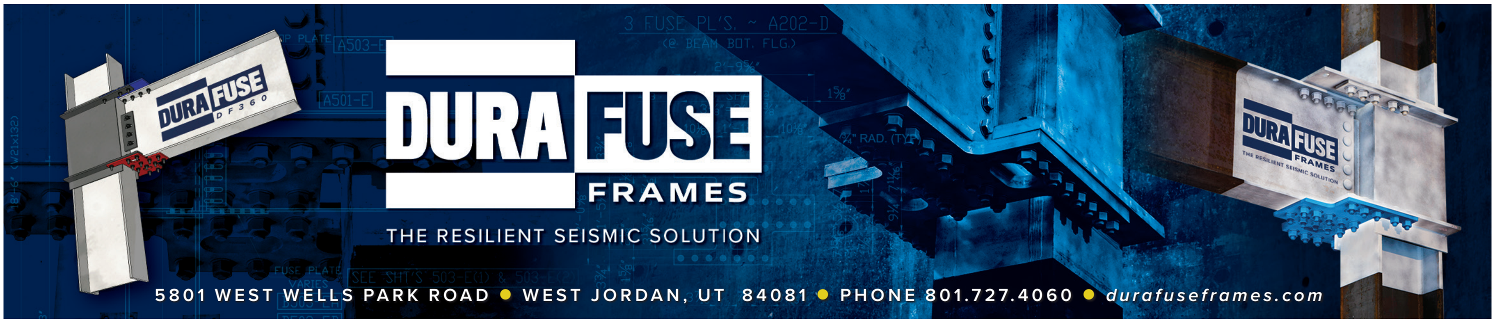

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TECHNICAL BULLETIN 22

Impact of Importance Factor and Post-Yield Stiffness on SMF Weight and Residual Drift

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Abstract: Steel special moment frames (SMF) are susceptible to unrepairable residual drifts. Two strategies for controlling residual drifts are designing with a higher importance factor and/or using SMF with higher post-yield stiffness. To investigate these two strategies, four-story SMF were designed considering different importance factors (1.0 or 1.5) and different post-yield stiffness [typical reduced beam section (RBS) or typical DuraFuse Frames (DFF)]. The SMF were analyzed with response history analysis (RHA) using eleven ground motions, scaled to design earthquake (DE) and maximum considered earthquake (MCE) levels. Designing with the higher importance factor increased weights 79-96%, but did not reduced residual drifts enough to ensure repairability for the I=1.5 RBS after DE loading. In contrast, designing with higher post-yield stiffness (DFF) had significant impact on residual drifts, with DFF having up to 34% lower residual drifts than RBS designed with the same importance factor. In addition to reduced residual drifts, DFF were 10-18% lighter and had up to 12% lower floor accelerations than comparable RBS frames.

Background

Steel special moment frames (SMF) are popular because they have excellent ductility and can accommodate flexible floorplans. Since SMF are designed using a high R-factor (8), significant inelastic behavior is expected during the design earthquake (DE) (AISC 2016b). In order to control inelastic deformations of SMF, drift limits are imposed during design (ASCE 2016).

There is justified concern that SMF may not be repairable after DE loading. Studies indicate that residual drifts for code-compliant SMF will be between 0.5% and 1.2% during DE loading (Erochko et al. 2011). Unfortunately, residual drifts need to be less than 0.5% to enable a building to be functional and repairable (Ariyaratana and Fahnestock 2011; McCormick et al. 2008) and residual drifts greater than 1.0% are considered a life-safety hazard and a total loss (ATC 1997; HBR 2020). Since code-compliant SMF are expected to have residual drifts beyond 0.5% under DE loading, they will likely not be repairable.

Two strategies for controlling residual drifts are: designing with a higher importance factor, and/or using SMF with higher post-yield stiffness. SMF for hospitals and other critical structures are designed with an importance factor (I) of 1.5 to improve performance and reduce residual drifts (ASCE 2016). However, numerous studies have demonstrated that a better approach for reducing residual drift is to increase the post-yield stiffness of the system (Pettinga et al. 2007). Residual drifts are "significantly more dependent" on post-yield stiffness than on building period, soil type, or system strength (MacRae and Kawashima 1997).

DuraFuse Frames (DFF) were developed in response to the SMF repairability problems discussed above. DFF have higher post-yield stiffness because of the shear yielding mechanism of the DFF fuse plates. The post-yield stiffness of a DFF connection (around 15% of initial stiffness) is ideal for residual drift control (Pettinga et al. 2007) and much greater than the 2-5% post-yield stiffness exhibited by connections that rely on beam yielding (eg. RBS). DFF are also easier to repair, if it ever becomes necessary, because all inelasticity is moved out of the beams and into replaceable fuse plates. DuraFuse Frames (DFF) are considered repairable for residual drifts of up to 2.5% (HBR 2020).

The purpose of the present study was to quantify the impact of importance factor and post-yield stiffness on SMF design and performance. Four-story SMF were designed considering different importance factors (1.0 or 1.5) and different post-yield stiffness [typical reduced beam section

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(RBS) or typical DuraFuse Frames (DFF)]. Response history analysis (RHA) was performed on the four-story frames using 11 ground motions, scaled to DE and maximum considered earthquake (MCE) levels. Results from the RHA were used to compare the performance of the various designs.

Building Characteristics and SMF Designs

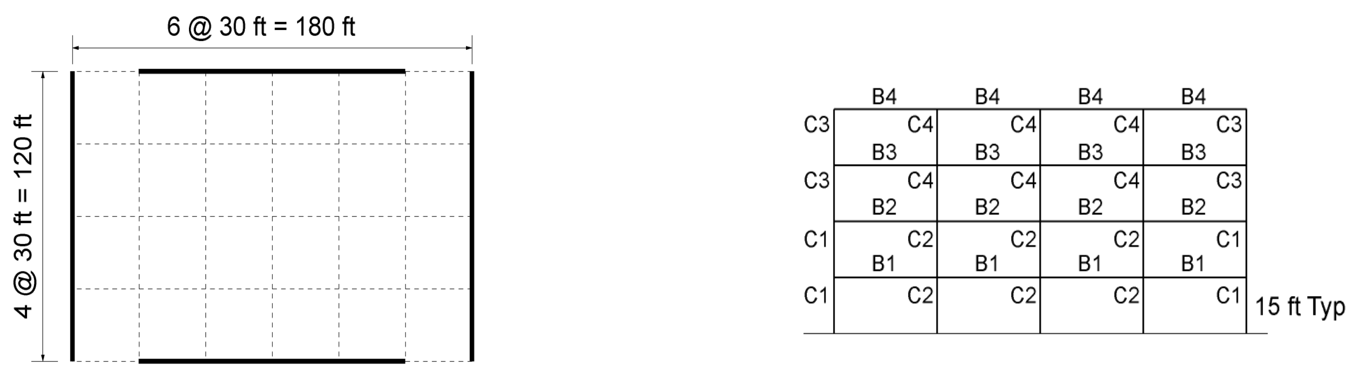
The prototype structure was a 4-story building, 180ft × 120ft in plan, with a story height of 15 ft (Fig. 1). The same seismic weight was used for the floors and roof (90 psf), assuming heavy rooftop equipment. The total seismic weight of the building was 7776 kips. The seismic force resisting system (SFRS) consisted of two SMF in each direction that were (4) bays wide [Fig. 1(a)].

The structure was designed for a Los Angeles site with S_{MS} = 2.0, S_{M1} = 1.35, S_{DS} = 1.33, S_{D1} = 0.9. Frames were designed using response spectrum analysis (RSA) in RAM Structural System (Bentley 2021) and optimized to meet drift requirements. SMF were designed per AISC 358 (2016a) and AISC 341 (2016b), including strong-column weak-beam requirements and other seismic checks.

Four designs were generated, considering two different values of the importance factor (1.0 and 1.5) in combination with two different SMF systems, reduced beam section (RBS) and DuraFuse Frames (DFF). Frames were designated RBS1.0, DFF1.0, RBS1.5, and DFF1.5. All frames were optimized to meet drift and strength requirements (ASCE 2016), while using W24× columns and beams no deeper than W36×. Table 1 summarizes the designs (with terms defined in Fig. 1).

Table 1 also summarizes the weight of each design (total for all frames in the building). Designing for I=1.5 had significant impact on weight. RBS1.5 was 96% heavier than RBS1.0 and DFF1.5 was 79% heavier than DFF1.0. The weight increase was greater than the 50% increase in I because the frames were drift controlled and the heavier W24× and W36× shapes for the I=1.5 designs were less efficient (stiffness per pound) than the lighter shapes that worked for the I=1.0 designs (this was particularly true for RBS1.5) (Table 1).

DFF designs were lighter than the RBS. DFF1.0 was 10% lighter than RBS1.0, while providing the same stiffness (Table 1). DFF1.5 was 18% lighter than the RBS1.5 while meeting the same stiffness requirements (Table 1).



(a) *(b)*

Fig. 1. *Prototype structure: (a) plan; (b) elevation of typical SMF.*

Table 1. *SMF designs for the study (terms defined in Fig. 1).*

Design	B1	B2	B3	B4	C1	C2	C3	C4	Weight (kips)*
RBS1.0	W30x108	W30x108	W24x76	W24x76	W24x131	W24x176	W24x103	W24x131	344
DFF1.0	W30x99	W30x99	W24x68	W24x62	W24x94	W24x131	W24x76	W24x103	308
RBS1.5	W36x210	W36x210	W36x182	W27x94	W24x229	W24x370	W24x176	W24x306	674
DFF1.5	W36x170	W36x194	W36x135	W27x84	W24x176	W24x250	W24x94	W24x192	550

*Weight as reported by RAM includes beams, columns, and DFF coverplates. It does not include RBS connection plates (doubler plates, continuity plates) or other DFF connection plates (fuse plates, top plates, bars). The weights of neglected connection plates for RBS and DFF would be comparable to each other.

Modeling Techniques and Validation

Models for individual frames were created in OpenSees (Mazzoni et al. 2006) to facilitate response history analysis (RHA). Beams and columns were represented with Timoshenko beam-column elements with lumped plasticity springs at the ends. Lumped plasticity spring properties for RBS and DFF were based on published guidelines (ASCE 2017; McCall and Richards 2020). Panel zone stiffness was represented with scissor springs with properties from published guidelines (Charney and Marshall 2006).

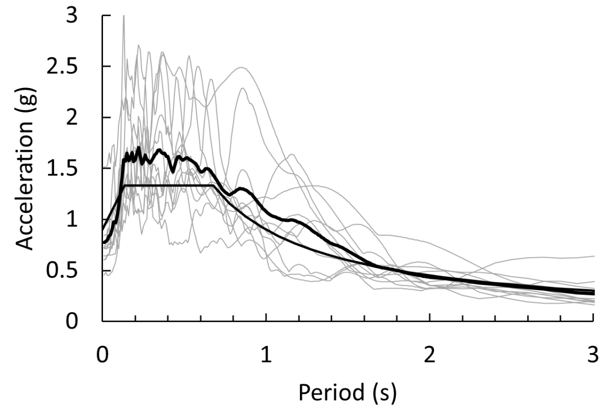
A leaning column was used to include the appropriate gravity forces for p-delta effects. A downward force of 972 kips at each level was applied to the leaning column prior to RHA to represent the gravity loads associated with one frame. Rayleigh damping was included to represent inherent damping, with parameters defined to give 2% damping at the first and third natural periods. The first natural periods for the RBS1.0, DFF1.0, RBS1.5, and DFF1.5 models were 1.48 s, 1.49 s, 0.88 s, and 0.95 s, respectively. Details on the modeling techniques and the applied masses and loads are provided in the Appendix.

Models were validated through eigenvalue analysis, pushover analysis, and lumped plasticity spring evaluation. Details on the validation are provided in the Appendix.

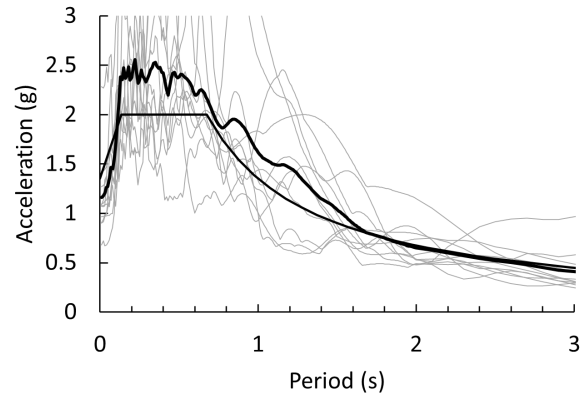
Ground Motions and Scaling

Ground motions from the P695 Far-Field Record set (ATC 2009) were used for analysis. Since RHA was performed on individual frames, individual components of records were considered, rather than pairs. Records were selected to minimize the required scale factors and provide good fit to the design spectra. The eleven records considered for the study and the scale factors used are summarized in Table 2. Different scale factors were used to represent DE and MCE loading.

Fig. 2 compares the spectra of the scaled records with the design spectra. The average spectra from the scaled records meet the requirements of ASCE 7 16.2.3.2. (ASCE 2016).



(a)



(b)

Fig. 2. *Scaled earthquake spectra: (a) for the DE; (b) for the MCE*

Table 2. *Ground motions used for response history analysis*

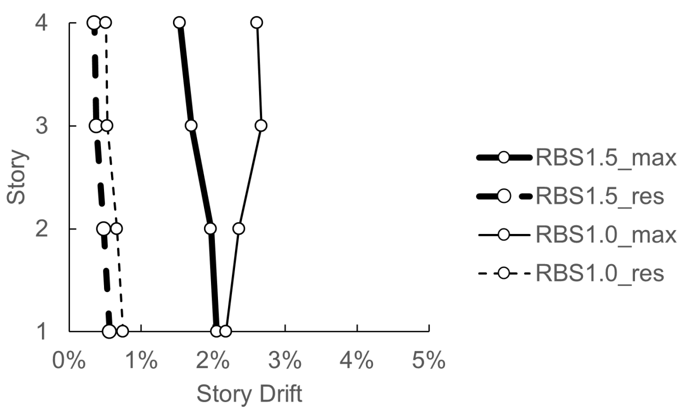
GM No.	Event	Recording	PEER-NGA Record No.	Scale Factors DE	Scale Factors MCE
1	Northridge, 1994	NORTHR/MUL009	953	1.68	2.52
2	Northridge, 1994	NORTHR/LOS000	960	1.76	2.65
3	Duzce, Turkey, 1999	DUZCE/BOL090	1602	1.81	2.72
4	Hector Mine, 1999	HECTOR/HEC000	1787	3.56	5.33
5	Imperial Valley, 1979	IMPVALL/H-DLT262	169	2.48	3.71
6	Kobe, Japan, 1995	KOBE/SHI000	1116	2.65	3.98
7	Kocaeli, Turkey, 1999	KOCAELI/DZC270	1158	2.00	3.01
8	Landers, 1992	LANDERS/YER360	900	2.95	4.42
9	Superstition Hills, 1987	SUPERST/B-ICC000	721	2.02	3.02
10	Superstition Hills, 1987	SUPERST/B-POE360	725	2.81	4.23
11	Chi-Chi, Taiwan, 1999	CHICHI/CHY101-E	1244	1.69	2.55

DE Results

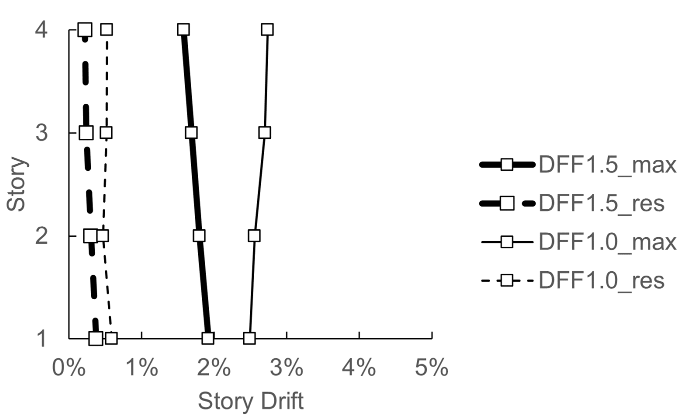
The key results from the response history analysis (RHA) were story drifts, residual story drifts, and floor accelerations. Average results for these response parameters were obtained for each story.

Results from the DE loading are shown in Figs. 3 and 4. For RBS1.0, the DE story drifts were around 2.5% rad [Fig. 3(a), light solid line], similar to the design intent (ASCE 2016). For RBS1.0, residual drifts ranged from 0.51% to 0.75% rad [Fig. 3(a), light dashed line], exceeding the 0.5% threshold (McCormick et al. 2008). The maximum and residual drift results for RBS1.0 were consistent with those reported by Erochko et al. (2011).

Results for RBS1.5 [Fig. 3(a), heavy lines] show the impact of designing with higher importance factor. By designing for I=1.5, story drifts were reduced to 2% or less, and residual drifts came down a little [Fig. 3(a), heavy dashed line]. However, residual drifts were still 0.6% at the base, exceeding the 0.5% threshold. These results demonstrate that designing for I=1.5 did not solve the SMF residual drift problem, despite increasing the frame weight by 96%.



(a)



(b)

Fig. 3. *Drifts from DE loading: (a) RBS frames; (b) DFF frames.*

Fig. 3(b) shows the DFF frames subjected to the same DE loading. DFF1.0 had residual drifts from 0.47% to 0.59% and DFF1.5 had maximum residual drifts of 0.37%, well below the 0.5% threshold. The maximum residual drift for DFF1.5 was 34% less than for RBS1.5. In fact, DFF1.0 had similar residual drifts to RBS1.5 [comparing Fig. 3(a) and (b)], despite RBS1.5 being 220% heavier (Table 1). These results, consistent with the literature, clearly demonstrate that "the post-yield stiffness of a system is the most important parameter controlling the magnitude of residual displacement"(Pettinga et al. 2007).

Fig. 4 shows the floor accelerations (absolute) for the various designs under the DE loading. The frames designed with I=1.5 (heavy lines) had 15-23% higher accelerations at the top than the I=1.0 designs. The DFF frames had lower roof accelerations than the corresponding RBS frames, with acceleration reductions of 8-12%

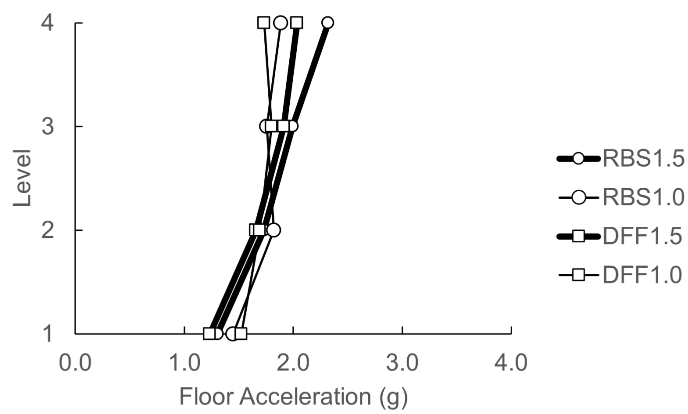
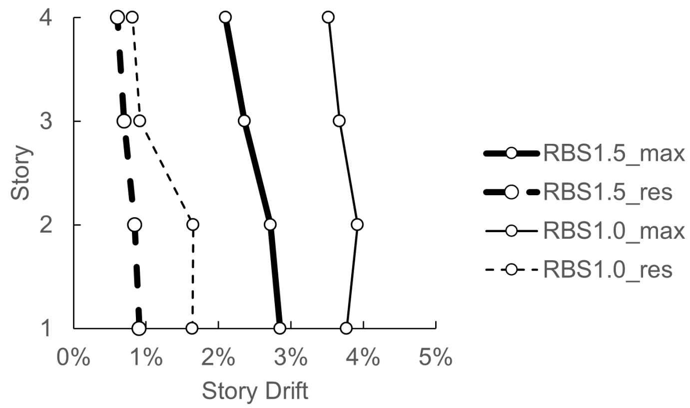


Fig. 4. *Floor accelerations from DE loading.*

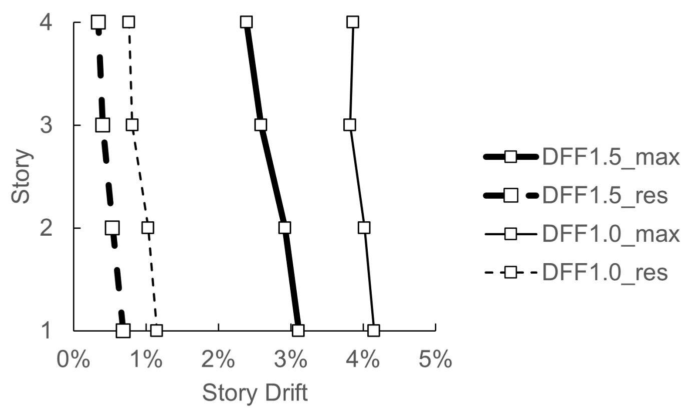
MCE Results

Under the MCE loading, RBS1.0 failed (could not converge) for two of the earthquakes (#7 and #11). This is unacceptable per Section 16.4.1 of ASCE 7 (2016), however for the purpose of discussion, results are presented based on the nine earthquakes where the solution converged. DFF1.0 could not converge for one of the MCE earthquakes (#11), which is permitted by ASCE 7 (2016). For the purpose of comparison, DFF1.0 results are shown for the same nine earthquakes that the RBS1.0 was able to pass. The I=1.5 designs (RBS1.5 and DFF1.5) had acceptable responses for all eleven MCE earthquakes, and the results shown for them are the average of all earthquakes.

Fig. 5(a) shows the response of RBS1.0 and RBS1.5 to MCE loading. Drifts were higher than for DE loading, with maximum story drifts of about 4% for RBS1.0 and 3% for RBS1.5. Residual drifts for both cases indicated frames with severe damage. For RBS1.0 and RBS1.5, the residual drifts of 1.6% and 0.9%, respectively, would be beyond repair [Fig. 5(a)]. The I=1.5 design reduced residual drifts, but not enough to fall below the repair limitation.



(a)



(b)

Fig. 5. *Drifts from MCE loading: (a) RBS frames; (b) DFF frames.*

Fig. 5(b) shows the response of DFF1.0 and DFF1.5 to the same MCE loading. As with the comparable RBS frames, the maximum story drifts were around 4% for DFF1.0, and about 3% for DFF1.5. But the residual drifts were lower for the DFF. The residual drifts for DFF1.0 and DFF1.5 were well within the repairable range for DFF [up to 2.5% (HBR 2020)].

Fig. 6 shows floor accelerations from the MCE loading. The frames designed with I=1.5 had 9-17% higher accelerations at the top than the I=1.0 designs. The DFF frames had lower roof accelerations than the corresponding RBS frames, with reductions of 6-12%.

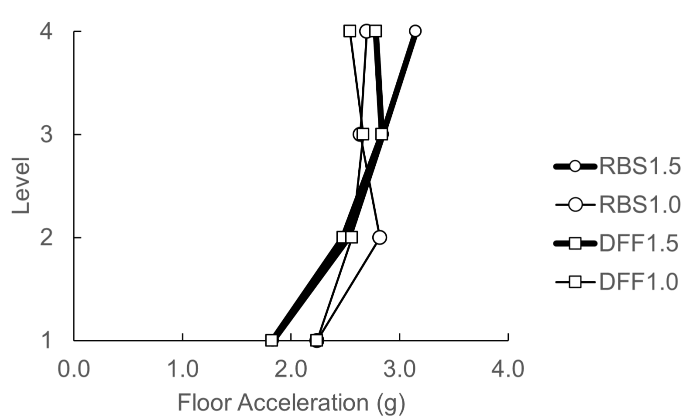


Fig. 6. *Floor accelerations from MCE loading.*

Conclusions

The purpose of the study was to investigate the impact of importance factor and post-yield stiffness on SMF weight and residual drifts. The results from the study support the following conclusions:

- The importance factor had tremendous impact on weight but minimal impact on residual drift (for DE loading). Comparing I=1.0 and I=1.5 RBS designs, weight increased 96% for I=1.5, while DE residual drifts remained unacceptable.
- Designing for an importance factor of 1.5 resulted in 15-23% higher roof accelerations (for DE loading).
- Increased post-yield stiffness, through the use of DuraFuse Frames, resulted in significant reduction of residual drift. Residual drifts for DFF were up to 34% less than comparable RBS (for DE loading). DFF designs (both I=1.0 and I=1.5) were at the 0.5% residual drift threshold for DE loading, and were repairable under MCE loading.
- The use of DFF also had positive impacts on weights and floor/roof accelerations. DFF frames were 10-18% lighter than RBS frames designed to the same importance factor, and had 6-12% lower roof accelerations (for DE loading).

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Appendix A – Modeling Techniques and Validation

Elements

Fig. A-1 illustrates the elements at a typical joint in the models. Beams and columns were represented with elastic Timoshenko beam-column elements (Mazzoni et al. 2006) with lumped plasticity springs at the ends to represent hysteretic behavior.

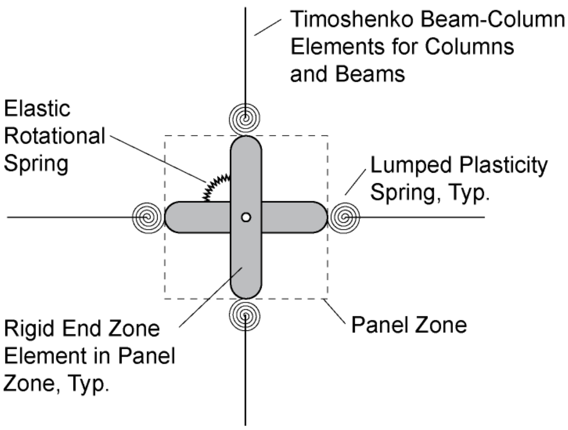

Fig. A-1. *Elements at a typical joint in the model*

An effective flexural stiffness was used for the beams and columns, so that less-than-infinite stiffness could be assigned to the rotational springs (Zareian and Medina 2010). For determining the shear stiffness of the beam and column elements, the shear area was $(d\text{-}t_f)t_w$ where d was the shape depth, t_f was the flange thickness, and t_w was the web thickness. An effective moment of inertia (90% of actual) was used for the RBS beams to represent stiffness reduction from the cutouts (Hamburger et al. 2009).

The lumped plasticity springs at the ends of the beams were defined using the MODIK material model in OpenSees (Ibarra et al. 2005; Lignos and Krawinkler 2011). Table A-1 summarizes the parameters for the MODIK model that were used. The parameters for the RBS were based on guidance from (ASCE 2017), and strengths were adjusted to reflect that the spring was positioned in the model at the face of the column. The parameters for the DFF were based on prequalification testing data (McCall and Richards 2020).

Table A-1 . *Parameters for the MODIK material model used for beam lumped plasticity springs*

Parameters	RBS	DFF
as_Plus, as_Neg	calculated	calculated
My_Plus, My_Neg	0.8	0.6
Lamda_S through Lamda_K	1000	1000
c_S through c_K	0	0
theta_p_Plus, theta_p_Neg	0.04	0.045
theta_pc_Plus, theta_pc_Neg	0.005	0.02
Res_Pos, Res_Neg	0.2	0.9
theta_u_Plus, theta_u_Neg	0.06	0.08
D_Plus, D_Neg	1	1
M_peak_Plus, M_peak_Neg	1	1

The lumped plasticity springs in the columns, where less plasticity was anticipated and realized, were modeled with steel01 material (Mazzoni et al. 2006) with 2% strain hardening.

Panel Zones

Beam elements with "rigid" properties were used to define the panel zone geometry (Fig. A-1). Since the panel zones were designed to preclude inelastic behavior, elastic representation of the panel zones was sufficient. Panel zone stiffness was represented with a scissor spring, with parameters determined per Charney and Marshall (2006)

Leaning Column

(Gravity Loads and Seismic Masses)

A leaning column was used to represent associated gravity framing and p-delta effects. The leaning column was pinned at the base and constrained to match displacements of the SMF at each floor level. The effective moment of inertia for the leaning column was 1904 in^4 for the bottom two stories and 756 in^4 for the top two stories, based on the weak-axis flexural stiffness of the associated gravity columns and orthogonal moment frame columns. A downward force of 972 kips at each level represented gravity loads associated with one frame (half the building). The seismic mass associated with each level for one frame was 2.52 $k\text{-}s^2/in$.

Natural Periods

As part of the OpenSees model validation, natural periods obtained from eigenvalue analysis of the OpenSees models were compared with those from the RAM models. Table A-2 summarizes results and shows general agreement between the independent models.

The similar periods for the DFF1.0 and RBS1.0 models (Table A-2) reflect that both were designed to the same stiffness criteria. While RBS1.5 and DFF1.5 were designed to the same drift requirements, RBS1.5 ended up with greater stiffness because of large jumps in section properties for heavy W24× and W36× sections. A lighter RBS1.5 design was not possible.

Table A-2 . *Results of eigenvalue analysis for OpenSees and RAM models.*

	First Mode Natural Period (sec)	
Design	OpenSees	RAM
RBS1.0	1.48	1.53
DFF1.0	1.49	1.53
RBS1.5	0.88	0.92
DFF1.5	0.95	0.96

The OpenSees models had slightly lower periods than the RAM models, indicating greater stiffness. For the RBS designs, this is primarily due to differences in panel zone modeling. The RAM model used a centerline method for representing panel zone flexibility (Bentley 2021). This method is known to be overly conservative for deep columns. The OpenSees models had more accurate panel zone representation (discussed above).

Pushover Analysis

As part of the OpenSees model validation, pushover analysis was performed. Fig. A-2 shows pushover curves for the various models. Since drifts tended to govern designs, and the RBS systems required heavier beams to meet drift limits (Table 1), the RBS designs had greater strength than the DFF designs with the same importance factor. Note, however, that the DFF designs performed much better (Figs. 3-6). The pushover analysis also demonstrates the higher post-yield stiffness for the DFF designs (Fig. A-2).

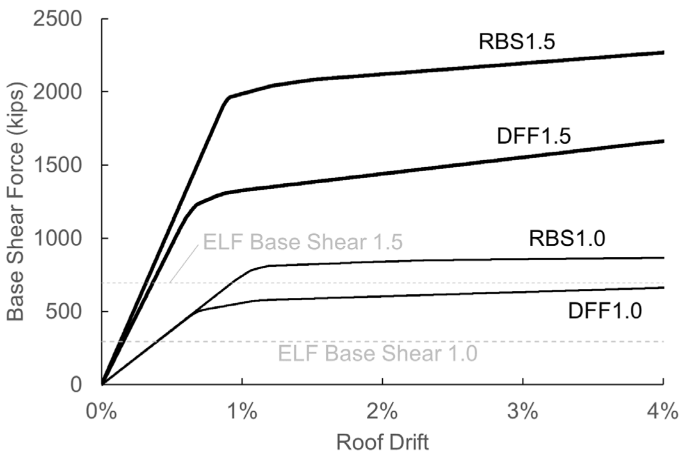

Fig. A-2 . *Pushover results from model validation study*

Sample Hysteretic Behavior

As part of the model validation, the beam springs from the models were investigated under the moment frame connection loading protocol (AISC 2016b). Fig. A-3 shows example results that demonstrate behavior consistent with the parameters listed in Table A-1.

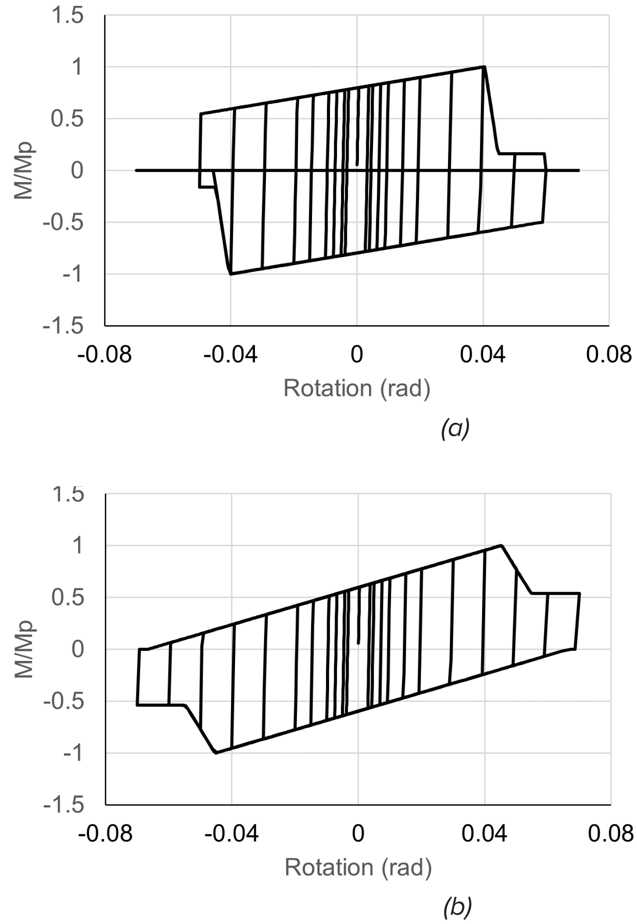

Fig. A-3 . *Lumped plasticity spring validation: (a) RBS, W30×108; (b) DFF, W30×99.*